Fatigue Resistance of Large High Tensile Steel Stay Tendons

Résistance à la fatigue des câbles de fils à haute résistance

Ermüdungswiderstand von Paralleldrahtkabeln grosser Tragfähigkeit

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SUMMARY
Large parallel wire stay tendons are being increasingly used in major bridge structures. This paper considers the various factors influencing the fatigue resistance of such stay tendons. In the case of HiAm and DINA tendons designed against fatigue according to the "limited damage" concept described in this paper it is possible to guarantee that all tendons in the construction remain intact and serviceable throughout their entire service life.

RESUME
Les câbles à fils parallèles de type HiAm et DINA de grande capacité portante sont de plus en plus utilisés pour la construction des ponts haubannés. Les différents facteurs qui influencent la résistance à la fatigue de tels câbles sont présentés dans cet article. Pour le dimensionnement à la fatigue de ces câbles on a adopté un concept de limitation des dommages, ce qui garantit que tous les câbles HiAm et DINA conservent toute leur capacité portante pendant la durée de vie de l'ouvrage.

ZUSAMMENFASSUNG
HiAm- und DINA-Paralleldrahtkabel grosser Tragfähigkeit sind in letzter Zeit vermehrt im Grossbrükkenbau verwendet worden. Im vorliegenden Beitrag werden die verschiedenen Faktoren beschrieben, welche den Ermüdungswiederstand solcher Grosskabel beeinflussen. Für die Bemessung auf Ermüdung wird ein Schadenbegrenzungs-Konzept vorgestellt, welches garantiert, dass sämtliche HiAm- bzw. DINA-Kabel einer Konstruktion während ihrer Lebensdauer die volle Tragkapazität beibehalten.

1. INTRODUCTION

During the course of the last decade there has been a considerable increase in the number of cable stayed structures which have been designed and erected. This form of construction has enabled the structural engineer to provide elegant solutions to large span construction at an economic cost. Figs. 7, 8 and 9 show examples of recently completed stayed bridges. One of the principal structural elements in such structures is the tensile stay which transmits the main girder loads to the supporting pylon. The types of stay tendons commonly used are built up from parallel strands or parallel wires or locked coil rope. During recent years there has been an increase in the use of high tensile steel parallel wire tendons in stayed structures.

Fig.1 shows a cross-section of a typical parallel wire stay-tendon used in bridge construction. The wire bundle consists of 7 mm dia. cold-drawn, patented high tensile steel wires with ultimate tensile strengths between 1600 and 1800 N/mm^2. Max. bundle sizes of 365 wires 7 mm dia. having an outside diameter of 200 mm have been realised. A robust 3-part protection system is provided through

- a film of anticorrosion fluid coated over the wires during the assembly of the wire bundle,
- a high-density polyethylene duct (wall thickness 8 mm to 11 mm) which also protects the tendon during transport and erection and permits the tendon to be wound on to a bobbin for delivery and erection purposes, and
- a resin-enriched cement grout injected into the polyethylene duct after the final force adjustment in the stay tendon.

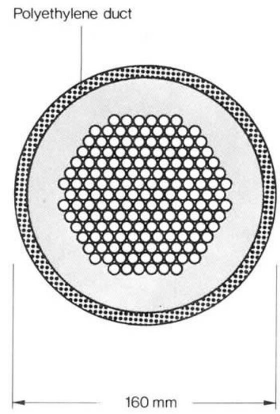


Fig.1

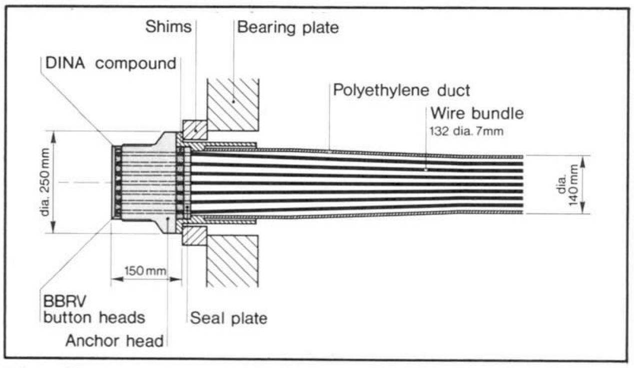


Fig.2

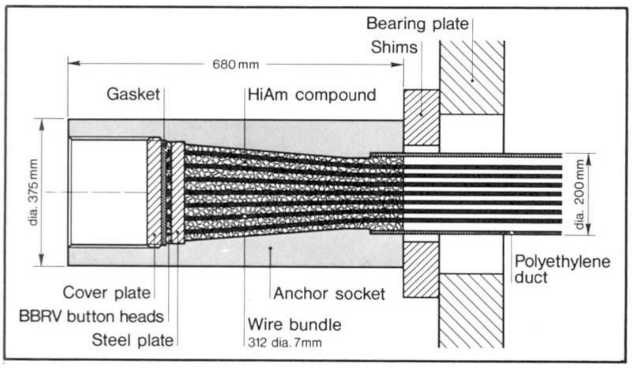


Fig.3

Figs. 2 and 3 show the BBRV-DINA and HiAm anchorages used for such stay tendons.

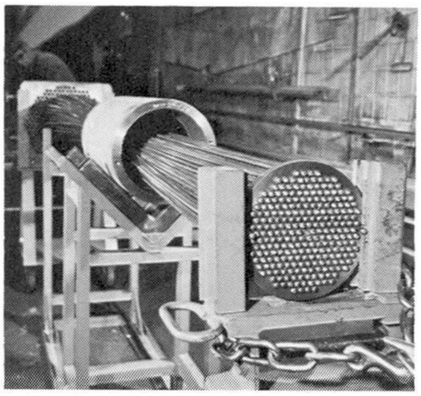


Fig.4

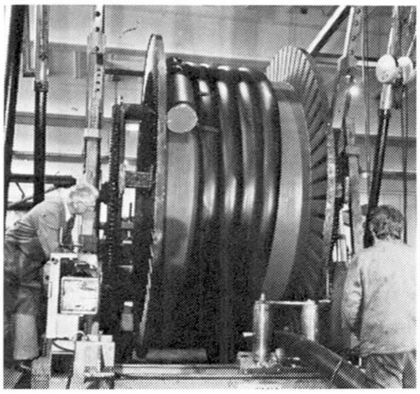


Fig.5


Fig.6

Figs. 4, 5 and 6 show various stages in the manufacture and erection of a HiAm stay tendon [1].

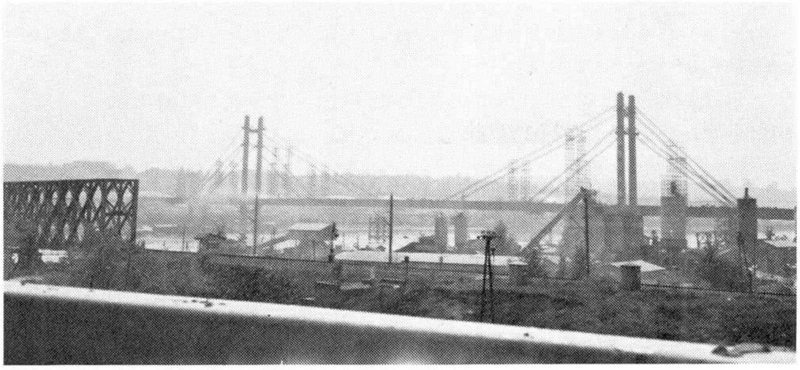


Fig.7

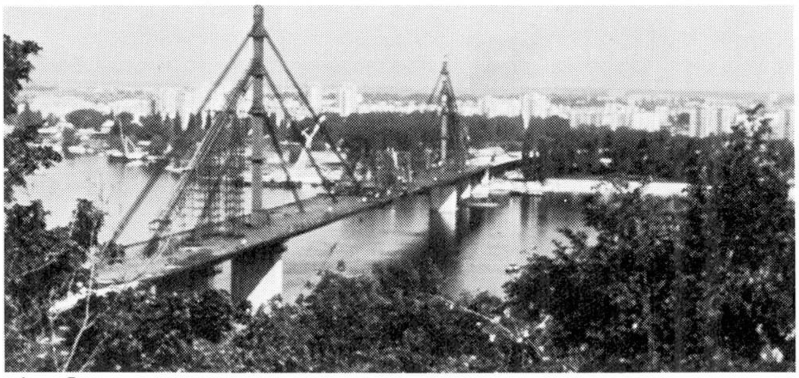


Fig.8

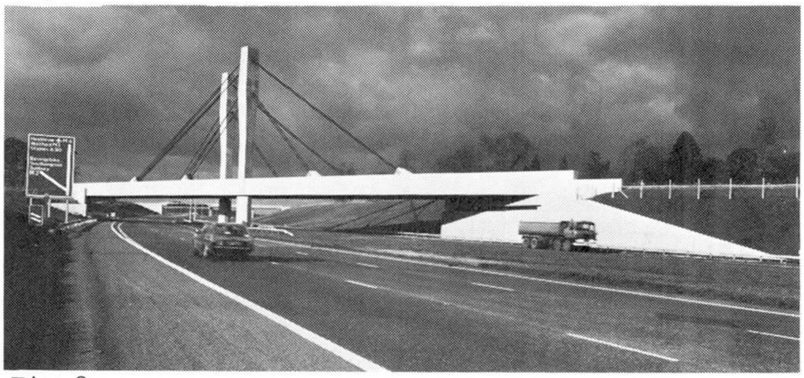


Fig.9

Tendon data for 3 stayed bridges recently completed in Europe are given below.

Save Bridge Belgrade, YUG [2], Fig.7:
for double track railway traffic. Steel box girder with main span of 254 m. Total 64 HiAm tendons clustered in groups of 4. Max. 290 wires 7 mm dia. per tendon. Max. length of tendon 116 m. Tendons arranged in two planes.

Danube Bridge Novi Sad, YUG (3), Fig. 8:
for double track road traffic. Steel box girder with main span of 351 m. Total 48 HiAm tendons clustered in groups of 4. Max. 312 wires 7 mm dia. Max. length of tendon 158 m. All tendons arranged in one plane.

Lyne bridge GB (4), Fig.9:
for double track railway traffic. Prestressed concrete girder with two spans of 54.9 m each. Total 16 DINA tendons with 79 wires 7 mm dia. Max. length of tendon 42 m. Tendons arranged in two planes.

Two typical features of this form of stayed construction are the large tensile forces directly transmitted by the stay tendons and the fluctuation of tensile forces in the tendons caused by the live load.

Table 1 shows the maximum and minimum tensile stresses in the stay tendons on the 3 bridges referred to earlier.

Bridge	Type	Size of tendon with max. stress variation	$\sigma_{max.}$ N/mm^2	$\sigma_{min.}$ N/mm^2	($\sigma_{max.} - \sigma_{min.}$) N/mm^2
SAVE (YUG)	Rail	4 x 260 dia.7 mm	450	229	221
NOVI SAD (YUG)	Road	4 x 208 dia.7 mm	703	447	256
LYNE (GB)	Rail	79 dia.7 mm	711	587	124

Table 1 Tensile stresses in bridge stay tendons

From these figures it can be seen that the fluctuations of tensile stresses in the stay tendons are quite significant. The stay tendons used for such applica-

tions should therefore not only be in a position to withstand large direct tensile forces but should also be fatigue resistant.
The designer is interested in the following properties of the stay tendons which play an important role in the choice of suitable tendon sizes.

- load-elongation characteristics of tendons
  In indeterminate structures like a stayed bridge, the tendon forces and structural deformations are a function of this characteristic. This characteristic is influenced by the type of wire bundle (i.e. whether parallel wire, parallel strand or locked coil rope), the tendon length and the stress in the tendon. The parallel wire tendon possesses an accurately predictable high value of Young's modulus E which remains constant over an unlimited number of loading and unloading cycles in the stress range encountered in stayed structures.
- ultimate strength of tendons composed of a large number of wires and the fatigue resistance of such tendons. The former property enables the engineer to decide on a suitable cross-section for the tendon so that the maximum tendon forces will be carried safely and the latter property enables him to ensure that at no time in the life of the structure will the repeated loadings cause distress in the tendons.

2. BEHAVIOUR OF PARALLEL-WIRE TENDONS UNDER STATIC AND FATIGUE LOADING

The question associated with these properties is how to accurately forecast and guarantee values for the static and fatigue resistance of stay tendons which could be as long as 200 m and as large as 365 wires 7 mm dia. in cross-section taking into account the numerous factors which can have an influence on these values.

2.1 Basic wire material

One of the most important factors affecting the static and fatigue resistance of tendons is the quality of the basic wire material used in their manufacture. The weight of wire used for tendons on a single project can be quite large. For example, 413 tons of cold drawn high tensile steel wire 7 mm dia. were used for the tendons on the Save Bridge, Belgrade. This material was delivered as 826 wire coils each of 500 kg weight. To assess the static and fatigue properties of such a large wire collective it is important to use the correct testing methods, to devise an adequate sampling procedure, and to employ suitable statistical methods for the evaluation of test results.
Static resistance:
For the Save Bridge project, a minimum of 2 specimens were taken from each wire coil and tested to determine the static tensile strength. In addition the yield stress and elongation after rupture were determined on a limited number of these specimens. Table 2 shows the results.

	Number of tests	Average	Standard deviation
Tensile strength (N/mm^2)	1800	1733	± 33
Yield (0.2 proof limit) (N/mm^2)	361	1548	± 39
Elongation after rupture δ_{10} (%)	361	8.1	± 0.5

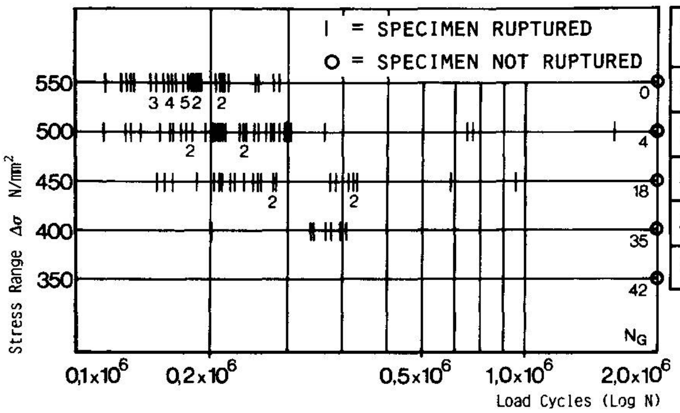
Table 2 Results of static tensile tests on wire

These results clearly show the uniformly high quality of this type of wire in respect of its static tensile strength properties.

Fatigue resistance:

In order to define the fatigue behaviour of wire completely it is necessary to possess information in the finite as well as infinite fatigue life range. This can then be represented in the form of a Wöhler curve relating the stress range $\Delta\sigma = 2\sigma_A$ to the number of load cycles N. Since fatigue test results display a large scatter it also becomes necessary to obtain sufficient information from tests to enable the various fractile Wöhler curves to be plotted. In the case of a large wire collective this implies that a large number of specimens have to be tested. There is however a practical limit on the number of test specimens which is imposed by the time to be spent on the tests and the costs incurred on testing. For the Save Bridge project 210 wire specimens were tested, which works out to roughly 1 specimen for every 4 wire coils used. The testing was organized to give a "complete random block". As shown in Fig.10 this was achieved by choosing 5 equally spaced values of stress range $\Delta\sigma = 2\sigma_A$ = 350, 400, 450, 500 and 550 N/mm^2 and testing 42 specimens at each of these values of stress range. Obviously the $\Delta\sigma$ values should be chosen in a range where fatigue failures will occur. Testing was done up to a maximum value of two million load cycles. It was assumed that practically infinite fatigue life is reached when a specimen endures the applied dynamic load for two million load cycles. Specimens which fail below this value of N are said to be in the finite fatigue life range. The values of N at which the specimens ruptured during the test are marked by the vertical dashes in Fig.10. The numerical values against the circles give the number of specimens which did not rupture during the test duration over $N_G = 2 \cdot 10^6$ load cycles. The large scatter of test results is evident from this figure and this characteristic is displayed by the very same wire whose static tensile characteristics are remarkably uniform.

Fatigue test results at 5 stress range levels with constant upper stress σ_o = 750 N/mm^2



n	r	r/n %	Pr %
42	42	100	98.4
42	38	90.5	89.0
42	24	57.1	55.9
42	7	16.7	15.7
42	0	0	0.8

$\Sigma n = 210$

l = specimen length = 200 mm
n = N° of specimens for each stress range $\Delta\sigma$
r = N° of ruptured specimens for $N_G \leqslant 2 \times 10^6$
P_r = Probability of rupture acc. to Weibull $= \frac{3r-1}{3n+1} \times 100$
(for r = 0, $P_r = \frac{1}{3n+1} \times 100$)

Fig.10

Figs.11 and 12 show the methods employed to derive the various fractile values for the endurance limit ($N_G = 2 \cdot 10^6$ cycles) and the fatigue strengths for finite life. These values have been used to plot the 5 % and 50 % fractile Wöhler curves shown in Fig.13. Ideally it would be desirable to have a Wöhler curve with Pr ≅ 0 which would define the absolute minimum fatigue strength for the wire collective. It is possible to estimate this value using the [arc. sin $\sqrt{Pr}$] Transformation. In the case of the wire used for the Save Bridge such a transformation yields a value of 340 N/mm^2 for the finite fatigue life strength ($N_G = 2 \cdot 10^6$ cycles) compared with the 5 % fractile value of 378 N/mm^2.

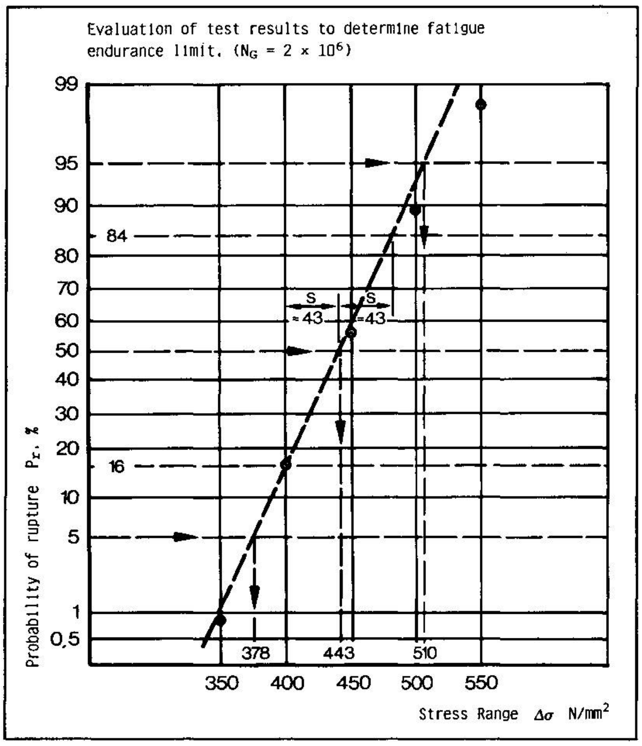


Fig.11

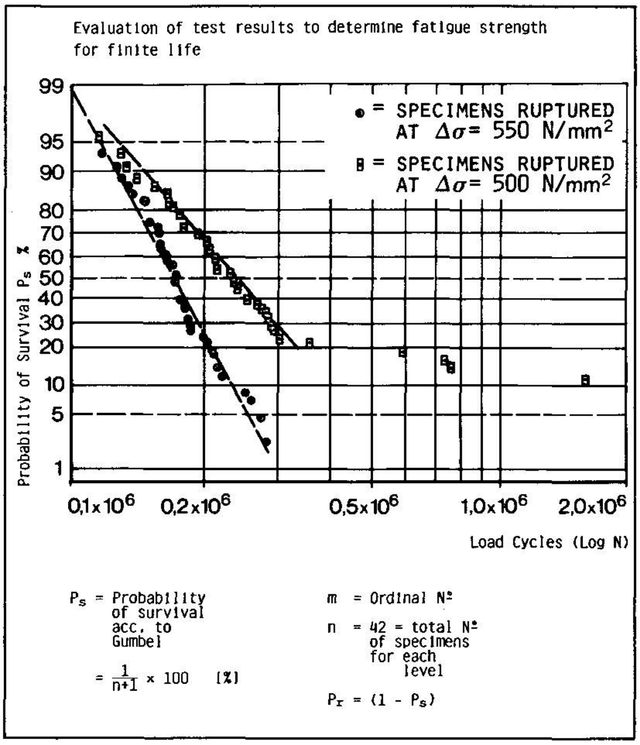


Fig.12

Cases may arise where only a limited number of specimens are available for testing. In such cases the "staircase" method of testing [5] may be adopted to obtain values of infinite fatigue life strength for the material tested (N_G = 2•10⁶ cycles). This method is however not as accurate as the complete random block method described earlier. Fig.14 shows the results of an evaluation of such a testing using 25 specimens for each of 2 different lengths l = 200 mm and l = 600 mm. The 2 lines tend to converge to a common point along the line Pr ≅ 0. For the same number of specimens tested the longer specimens include a greater amount of wire material and therefore the Δσ value for Pr = 50 % and the standard deviation S are smaller than the corresponding values for shorter specimens.

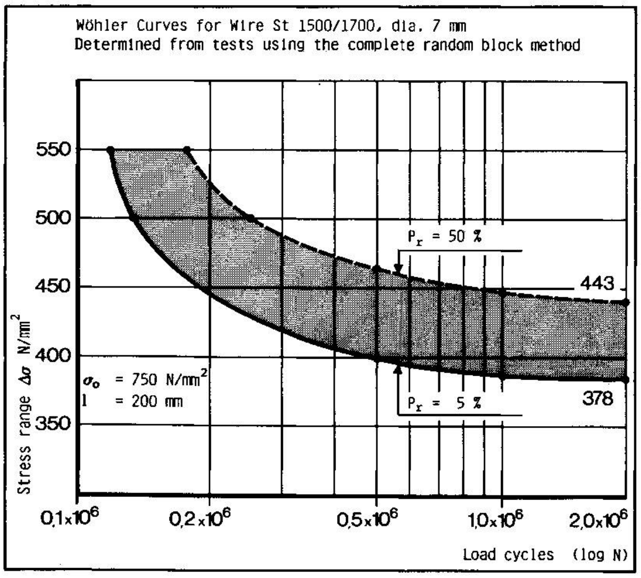


Fig.13

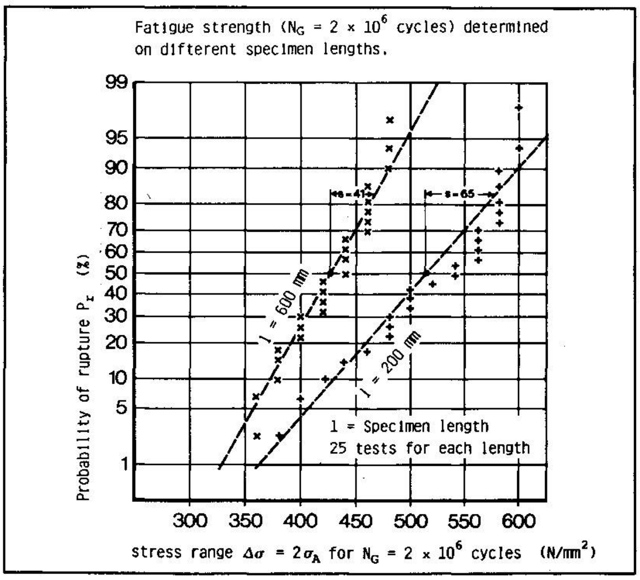
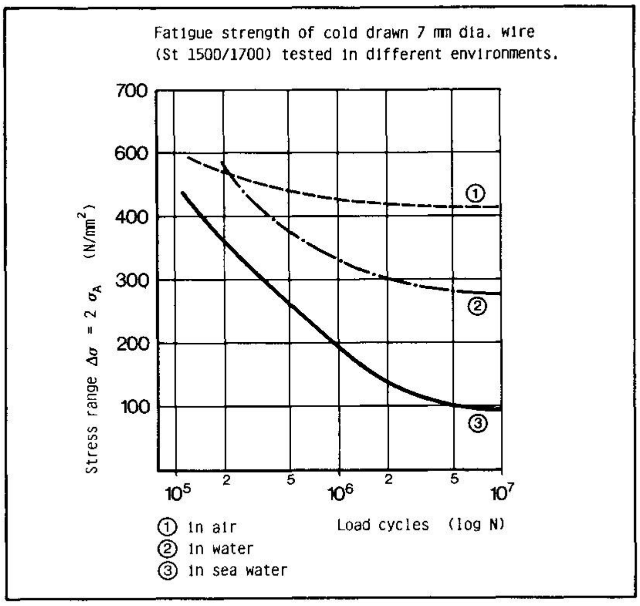


Fig.14

The fatigue resistance properties of wire described above were established from tests on 'as-drawn' wire specimens in a non corrosive environment. The necessity for an absolutely reliable system of protection against wire corrosion is evident from Fig.15 which shows the fatigue resistance of wires subject to different environments [6]. Specimens tested in sea water show a large reduction in fatigue resistance. The protection system described in an earlier part of this paper ensures that the wires in the tendon are protected from corrosive influences during service life.



In the search for a reliable corrosion protection system to tendons, the possibility of a primary protection by galvanizing the wire has also been considered. Recent tests on galvanized, cold-drawn high tensile steel St 1500/1700 wire specimens have yielded infinite fatigue life ($N = 2 \cdot 10^6$ cycles) at a stress range $\Delta\sigma = 2\sigma_A$ of at least 450 N/mm^2. These values lie well above those obtained with ungalvanized wire of the same quality.

Fig.15

2.2 Anchored tendon

In all static and dynamic tensile tests on tendons fitted with HiAm or DINA anchorages the wire failure occured in the free length of the tendon. The anchorages did not reduce either the static or the dynamic tensile strength of the tendons. These tendon properties are therefore primarly governed by the corresponding properties of the wire bundle alone. This is reflected in Table 3 which shows the results of static and fatigue tests carried out on various HiAm and DINA tendons. All tendon specimens were subjected to various fatigue loads as shown in the table. On conclusion of the fatigue loading, the tendon specimens were subjected to ultimate static tensile tests to establish the static breaking loads. It was thus possible to establish the damage D caused to the tendons by the foregoing fatigue loading. It is seen that the largest value of damage observed in these tests was 1.3 % or a loss of 3 out of 295 wires in a tendon. Even after $2 \cdot 10^6$ cycles of loading at $\Delta\sigma$ = 20 kp/mm^2 (200 N/mm^2) this tendon was in a position to carry 98.7 % of its static breaking load.

Disposition of Bundle					Fatigue Test					Breaking Test after Fatigue	
TYPE	NUMBER OF WIRES mm	STEEL GRADE kp/mm^2	AVERAGE TENSILE STRENGTH β_Z kp/mm^2	CALCULATED BREAKING LOAD Z_u Mp	UPPER STRESS σ_u kp/mm^2	AMPLITUDE $\Delta\sigma$ kp/mm^2	NUMBER OF CYCLES N	NUMBER OF FAILURES n	REDUCTION OF STEEL AREA %	MEASURED BREAKING LOAD Z'_u Mp	DAMAGE $\frac{Z_u - Z'_u}{Z_u}$ %
HIAM	295Ø7	140/160	169	1880	56	20	2×10^6	3	1.0	1856	1.3
HIAM	295Ø7	140/160	178	1980	59	20	2×10^6	1	0.3	1974	0.3
HIAM	19Ø7	150/170	179	128	60	20	2×10^6	0	0	128	0
DINA	102Ø7	150/170	179	688.6	60	20	2×10^6	0	0	688.6	0
DINA	55Ø7	150/170	175	363	110	25	2×10^6	0	0	363	0

Table 3 Results of static and fatigue tests on HiAm and DINA tendons

This is an important characteristic of tensile members composed of a multitude of identically sized and loaded wires. Even after fatigue loading under service, at the most only a small number of the component wires may fail and the member itself is still capable of withstanding pratically the entire original breaking load.

3. DESIGN ASSUMPTIONS

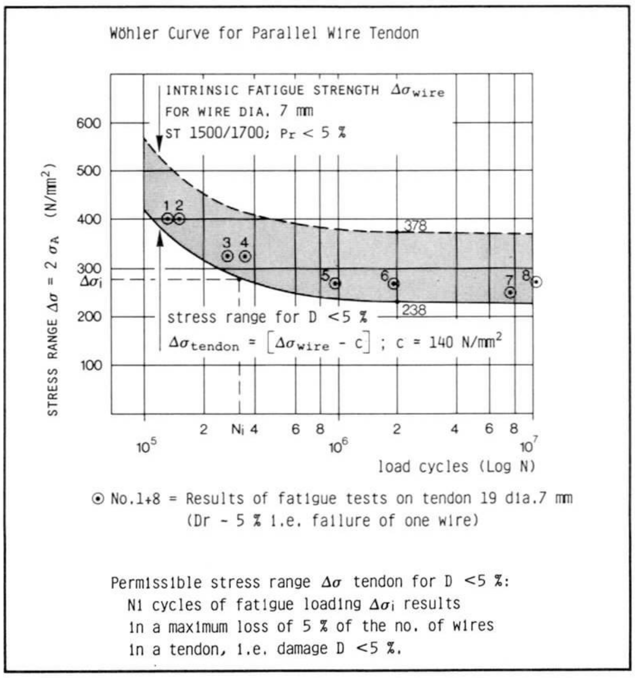
3.1 Static loads

The maximum stresses in the tendons max.σ caused by the design loads should be limited to the permissible value perm.σ laid down in the specifications. In some cases it may be necessary to consider additional stresses caused by angular variations of the tendon in the vicinity of anchorages. In the case of the Save Bridge, Belgrade [7] and the Lyne St. Bridge [8] the maximum stresses in the extreme wire caused by local bending amounted to 10-20 % of the axial stresses. Usual practice is to limit the tendon stresses caused by axial tensions to 0.45 of the ultimate tensile strength of the wire. For cold drawn high tensile steel wire quality St 1500/1700 shown in Table 2 this works out to 765 N/mm^2.
It should be noted that at stress levels of 0.45 β_Z in the wire the effects of creep can be neglected and that the tendon may be considered to behave elastically up to the design loads.

3.2 Fatigue loads

Stress variations $\Delta\sigma$, when repeated over a large number of load cycles, lead to a damage of the material so stressed. In the case of stay tendons the damage may sometimes result in a small number of wire breakages. The aim of designing against fatigue in tendons should be to avoid or to limit this damage to a very small value during the life time of a bridge so that the tendon remains largely intact even after being subject to the loading which caused the damage.
The designer will therefore need to have a curve relating the permissible stress range $\Delta\sigma = 2\sigma_A$ to the number of load cycles N for which the damage D $\leqq$ say 5 %. Such a curve is shown in Fig.16 and has been deduced from the 5 % fractile curve (intrinsic fatigue strength) for the basic wire material used for the tendon.

$\Delta\sigma_{Tendon} = [\Delta\sigma_{Wire} - c]$; $c \cong 140\ N/mm^2$



Any fatigue load ($\Delta\sigma$, N) on or below this curve will result in, at the most, a loss of 5 % of the number of wires in a tendon which can subsequently still resist at least 95 % of its original breaking load.
Results of a series of tests on short (l=3000 mm) HiAm tendons are also plotted on this figure. It can be seen that the value of c chosen for these tendons is adequate. It is worth noting in this context that tests on short tendons generally yield lower values of $\Delta\sigma$ since they are very sensitive to small manufacturing tolerances.

Fig.16

The present procedure when designing against fatigue is to determine the max. stress range max.$\Delta\sigma$ in a tendon and to read off the related number of load cycles N from the tendon Wöhler curve. This is compared with a specified value of N. However, this appears a severe condition to be checked for since the value of max.$\Delta\sigma$ is only a peak value in a load collective which occurs during the life time of a bridge.
Using the Palmgren-Miner hypothesis of linear accumulation of damage and the Wöhler curve for constant stress range it is possible to derive a fatigue life curve (max.$\Delta\sigma$, $\overline{N}$) for any desired load collective. Fig.17 shows how the Wöhler curve of Fig.16 drawn to a log-log scale can be represented by 2 straight lines. The curve can be described by the equations

$$N = N_D \cdot \left(\frac{\max.\Delta\sigma}{\Delta\sigma_D} \right)^{-k} \qquad (N < N_D = 10^6)$$

$$\Delta\sigma_D = \text{constant} \qquad (N > N_D = 10^6)$$

and is valid for a load collective p = 1.0, i.e. constant stress range max.$\Delta\sigma$.

A load collective p = 0.5 for a block of 10^6 cycles is also shown in Fig.17. The peak value max.$\Delta\sigma$ of the stress range is the same as for load collective p = 1.0, but occurs only once (10^0). The minimum value of the stress range is 0.5•max.$\Delta\sigma$. The procedure [9] for deriving the fatigue life curve for this load collective can best be described with reference to, say, the stress range level max.$\Delta\sigma$ = 350 N/mm^2. The load-collective is described by the continuous curve which dips from a maximum value of 350 N/mm^2 to a minimum value of 0.5 • 350 N/mm^2. This range between 350 and 175 N/mm^2 is divided into a convenient number of steps. For each of these steps the number of load cycles n_i is read off and the partial damage n_i/N_i calculated. Obviously $\Sigma n_i = 10^6$ cycles. The cumulative damage $S = \Sigma n_i/N_i$ related to the block of 10^6 cycles can then be calculated. The Miner condition $\overline{S} = 1.0$ is fulfilled by a value $\overline{N} = 10^6/S$ which gives a point (350, $\overline{N}$) on the fatigue life curve. This procedure is repeated for other values of max.$\Delta\sigma$.

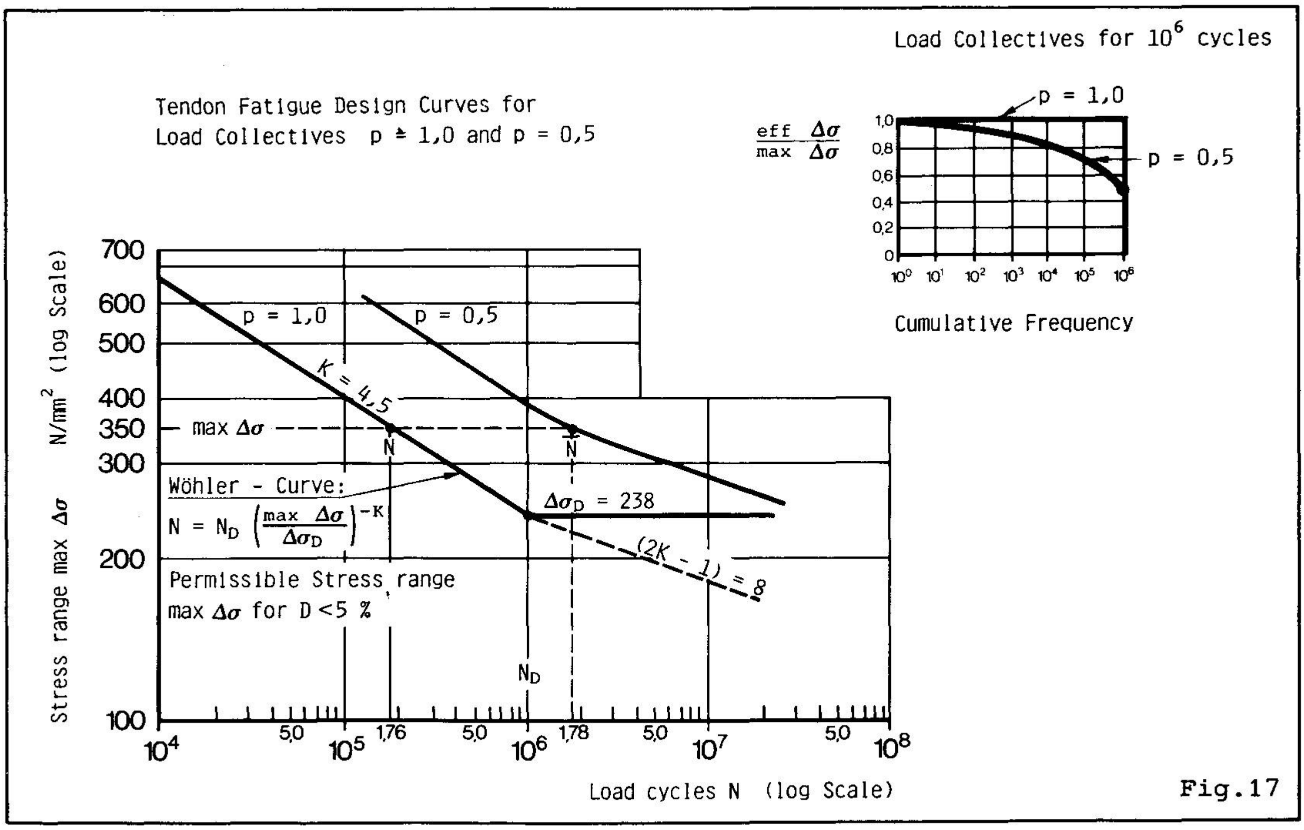


Fig.17

The effect of stress ranges lying below the endurance limit value $\Delta\sigma_D$ has been included by using the fictitious curve shown dashed according to the procedure suggested by Haibach [10]. The fatigue life curve (max.$\Delta\sigma$, $\bar{N}$) shows that for the load collective p = 0.5 shown in Fig.17 values of $\bar{N}$ are obtained which lie considerably above the values of N read off the Wöhler curve p = 1.0.

4. CONCLUDING REMARKS

It is well-known that fatigue loading causes damage to the material on which it acts. When designing stay tendons against fatigue, the principle of limiting such a damage in each tendon to a small value should be adopted and the tendon construction so chosen that a substantial load carrying capacity is still available in it even after the damage has occurred.

The parallel-wire HiAm and DINA tendons fulfil these requirements admirably in that each tendon is composed of a multitude of wires carefully chosen for their high intrinsic fatigue resistance. Using the limited damage concept outlined in this paper it is possible to guarantee that none, or at the most, only a very small number of wires in any tendon cross-section become ineffective under fatigue loads and the tendon as a whole remains intact during the life time of the structure.

REFERENCES

[1] Birkenmaier, M.: Fatigue resistant tendons for cable-stayed construction. IABSE Proceedings P-30/80, May 1980

[2] Hajdin, N., Jevtovic Lj.: Eisenbahnschrägseilbrücke über die Save in Belgrad. Der Stahlbau 47 (1978), Heft 4.

[3] Hajdin, H.: Highway Bridge "23 October" over the Danube at Novi Sad. Proc. 6th Yugoslav Congress of Structural Engineers, Bled 1978. (in Serbo-Croatian).

[4] Concrete Society: All-concrete cable-stayed railway bridge. Concrete Vol. 13, April 1979, No.4.

[5] Deubelbeiss, E.: Dauerfestigkeitsversuche mit einem modifizierten Treppenstufenverfahren. Materialprüfung 16 (1974), No.8.

[6] Nürnberger, U., Wiume, D.: Möglichkeiten des Korrosionsangriffes bei Seilen und Bündeln. Berichtsband aus der Vortragsveranstaltung Seile und Bündel im Bauwesen, Haus der Technik, Essen. September 1981.

[7] Kollbrunner, C. F., Hajdin, N., Stipanic, B.: Contribution to the analysis of cable-stayed bridges, No.48. Institut für bauwissenschaftliche Forschung Stiftung Kollbrunner/Rodio, Zürich, November 1980.

[8] Private communication from K. Kretsis, October 1981.

[9] Schütz, W.: Lebensdauer-Berechnung bei Beanspruchungen mit beliebigen Last-Zeit-Funktionen. VDI Berichte Nr. 268, 1976.

[10] Haibach, E.: Modifizierte lineare Schadensakkumulations-Hypothese zur Berücksichtigung des Dauerfestigkeitsabfalls mit fortschreitender Schädigung. LBF-TM Nr. 50/70.